BROHM MINING CORP.

GILT EDGE MINE

GEOTECHNICAL ASSESSMENTS AND PRELIMINARY SLOPE DESIGN ALTERNATIVES

December, 1988

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GEOTECHNICAL AND
HYDROGEOLOGICAL CONSULTANTS

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Prepared by

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1. INTRODUCTION

This report summarizes the investigation and analyses carried out to prepare preliminary alternative open pit slope designs for the Gilt Edge Mine. The terms of reference for the study were summarized in a letter proposal dated October 6, 1988 from Piteau Associates Engineering Ltd. to Mr. T. Fox of Brohm Mining, and were discussed in various telephone conversations in October, 1988. Authorization to proceed was given in a letter from Brohm Mining dated November 7, 1988. Results of the investigation, analyses and design recommendations were summarized in a fax transmission to Mr. T. Fox dated December 30, 1988.

Assessments of the engineering geology, structural geology and rock competency are presented in Section 3. Slope stability analyses and alternative slope designs are presented in Section 4, while conclusions and recommendations are summarized in Section 5.

2. DESCRIPTION OF THE INVESTIGATION

2.1 FIELD INVESTIGATION

Mr. A.F. Stewart of Piteau Associates Engineering Ltd. (PAEL) visited the site from December 4 to 8, 1988. During this time, background information concerning the general geology, mineralogy, geologic structure, groundwater, etc. was obtained from mine personnel along with copies of relevant plans, geologic sections, etc. Inspection of the mine area and geologic structural mapping and documentation of existing rock slopes was carried out. Diamond drill core logs and photographs of selected diamond drill core were examined. A limited number of representative samples of the various rock types were obtained for point load index strength testing.

2.2 OFFICE STUDIES

To obtain approximate unconfined compressive strengths for the various rock units present at the mine, point load index tests were conducted. Shear strength characteristics of joint surfaces were estimated based on visual classification.

Geologic structural analyses and slope stability analyses were carried out using the limited amount of geologic data and strength testing results that were available. The geological mapping data were processed and plotted on lower hemisphere equal area projections using a desktop computer. Statistical methods were used to define the basic geologic structural parameters and to develop an appreciation of the nature and distribution of these features in the rock mass.

Based on the results of the structural analysis, and on a review of geologic sections, the pit was divided into areas wherein the general rock type and geologic structure are expected to be similar (i.e. structural areas). In order to develop alternative slope design criteria for a complete range of slope orientations, numerous design sectors were defined. Within each design sector, the geologic structure, rock strength and orientation of the pit slope are assumed to be similar. Separate slope stability assessments were carried out for each design sector. Equal area projections were used to determine the kinematically possible failure modes which are likely to control slope design in each design sector. Mechanical stability analyses were carried out on all possible failure modes and alternative slope design configurations were established. Slope design considerations were developed with due consideration of effects of strength, groundwater conditions etc., and with the object of providing sufficient information to prepare preliminary slope designs. Based on the configuration of two interim pits (i.e. end of Year 1 and end of Year 3) and one ultimate pit provided by Brohm Mining, preliminary interramp slope angles were prepared and summarized on plan.

3. ENGINEERING GEOLGOY

Detailed reports concerning the geology of the mine area have been prepared by R.T. MacLeod of Brohm Mining Corp. (see references in Section 7) and by Minproc Engineers Inc. Information pertinent to the slope stability study has been freely extracted from these descriptions and included in the discussion below.

3.1 REGIONAL SETTING

The Gilt Edge Mine is located in western South Dakota, approximately 5 miles southeast of Lead in Lawrence County. Gold was first produced in the mine area in the late 1880's and has been mined intermittently ever since.

The mine is situated in the northern Black Hills area at an elevation of about 5000 to 5500 feet. Topography is somewhat rounded and vegetation is moderate, consisting largely of coniferous trees. Average annual precipitation in the area is about 30 inches (combined rainfall and equivalent snowfall).

The Black Hills of South Dakota, Wyoming and Montana represent a north-northwest trending elongate dome that is approximately 60 miles wide and 120 miles long. Doming of this region occurred in Early Tertiary time, or about 60 to 65 million years ago. As a result of uplift and doming, rock units older than the age of uplift activity were tilted, and now radially dip away from the central core of the dome. Rock units exposed in the Black Hills region consist of Archean and Proterozoic igneous rocks, Precambrian metasedimentary rocks, sedimentary rocks ranging in age from Paleozoic to Mesozoic, igneous rocks of Cenozoic age and Quaternary sediments.

Igneous rocks of Cenozoic (Early Tertiary) age occur in a west-northwest trending belt approximately 12 miles wide and 50 to 60 miles long. The Gilt Edge Project area is located at one of the major sites of intrusive activity along this belt.

The Precambrian rock units in the Gilt Edge area generally strike northwest and dip to the southwest. Folds within these units usually plunge to the southeast. Regionally, the Deadwood Formation strata dip gently to the northeast. However, this pattern is disrupted locally by doming caused by intrusion of the Tertiary stocks.

Steeply dipping to nearly vertical, north, northeast and northwest trending faults and fracture zones are prevalent in the mine area. Northeast and northwest fault trends predominate, and in some cases are best described as fracture zones of more or less parallel, low-displacement offsets ranging in width from approximately 20 to 100 feet or more. The northeasterly trend is subparallel to the margins of the Early Tertiary igneous stocks and is believed to have been a major zone of weakness in the basement rocks. The emplacemnt of the Gilt Edge Tertiary stocks is controlled by the intersection of the local northeast trend with the regional northwest structure trend.

3.2 BEDROCK LITHOLOGY AND ALTERATION

Within the Gilt Edge Project area, multiple alkalic igneous rocks of Early Tertiary age intruded Precambrian metasedimentary and Cambrian sedimentary rock units (see Fig. 1). The main rock types of interest to this study that are exposed in this area include Precambrian metasedimentary rocks, Upper Cambrian Deadwood Formation rocks and Eocene intrusive rocks that have been subdivided into (from oldest to youngest), hornblende trachyte porphyry, trachyte porphyry and quartz trachyte porphyry. Of these, the Precambrian rocks, trachyte porphyry and quartz trachyte porphyry are by far the most abundant and thus of prime importance to this study.

The Precambrian rocks are composed of quartz mica schist with amphibolite and minor amounts of metachert. The Upper Cambrian Deadwood Formation consists of a basal quartzite that ranges in thickness from 0 to about 140 feet.

The oldest Tertiary intrusive rock exposed on site consists of green to grey hornblende trachyte porphyry. This unit forms laccolithic bodies, sills and dikes, and intrudes the earlier units. The hornblende trachyte is typically porphyritic with phenocrysts set into a microcrystalline groundmass of potassium feldspar.

The trachyte prophyry is light to medium gray, with cryptoperthite and minor plagioclase phenocrysts contained in a groundmass of microcrsystalline potassium feldspar. The quartz trachyte porphyry is light gray and consists of sanidine, cryptoperthite, plagioclase, and quartz phenocrysts in a groundmass of mostly potassium feldspar.

The hornblende trachyte unit generally forms a laccolithic body that intruded along the contact between Precambrian rocks and the Deadwood Formation. It also occurs within the Deadwood Formation, mainly as sill-like bodies that inflated the thickness of the Deadwood. Within Precambrian rock units, the hornblende trachyte intrusive bodies are more dike-like and tend to occur as intrusives in faults and along foliation within the Precambrian rocks. In contrast to the hornblende trachyte, the trachyte porphyry and quartz trachyte porphyry units occur as intrusive stocks in the Gilt Edge mine area. Outward from the mine area these units form small plugs, dikes and sills in the Precambrian and Cambrian country rocks.

Brecciated masses of rock occur marginal to the intrusive stocks of quartz trachyte prophyry. These breccias include variable size fragments of all rock types described above with the exception of the quartz trachyte porphyry. The breccia matrix is composed of quartz, clay and iron-oxide minerals in the oxidized zone. Where unoxidized at depth, the breccia is typically composed of quartz, pyrite, marcasite, fluorite, and clay minerals.

The most intensely altered rock unit in the mine area is the trachyte porphyry. Alteration observed within this stock includes argillic, sericitic and potassic suites of mineralization. In addition, prophyllitic and weak argillic alteration has been observed within the quartz trachyte prophyry. However, alteration does not seem to have been severe enough to significantly weaken the various rock units.

3.3 STRUCTURAL GEOLOGY

A rational slope stability analysis and slope design requires that the mine area be subdivided into areas of approximately similar geologic structural characteristics. The engineering behaviour of the slope forming materials can be expected to differ in areas of the rock mass which have different geologic structural characteristics. Hence, extrapolation of stability analysis results and slope design criteria is only valid within areas of the rock mass having similar geologic structural characteristics. Such areas with similar geological structure are called structural areas or structural domains.

At the Gilt Edge Mine, six basic stuctural areas coinciding with the six main rock units, were defined as follows:

1. Precambrian Schist
2. Trachyte Porphyry
3. Quartz Trachyte Porphyry
4. Trachyte Breccia
5. Hornblende Trachyte Porphyry
6. Deadwood Sediments

The need for further definition or refinement of structural areas may become apparent during mining when some of the pit slopes have been exposed and more mapping data than the presently limited amount is available.

Attitude of geological structures is the most important consideration in determining whether geological structural populations within a structural area, or between structural areas, are similar or dissimilar. Other parameters, such as continuity (joint extent or size), roughness, infilling, etc. were considered in evaluating the engineering properties and nature of the joint sets, but were not used for designation of structural areas. Lower hemisphere equal area projections were used to define the peak or average orientation for each main discontinuity set mapped in each structural area.

3.3.1 Joints

Attitudes of the main joint sets are summarized in Table I and shown on lower hemisphere equal area projections in Appendix A. Throughout the mine area, joint sets are generally seen as reasonably well defined clusters. In some cases, it is reasonable to represent each joint set by a single peak orientation. However, other joint sets are diffuse enough that they may be more readily represented by two or three different peaks. In such cases, the joint set is represented by a number of subsets (e.g. Joint Set A1, Joint Set A2, etc.).

As summarized in Table I, up to six joint sets were recognized in the mine area (i.e. Foliation joints and Joint Sets A to E). However, only three sets (i.e. Foliation Joints and Joint Sets A and B) are considered as being particularly continuous and well developed within either the Precambrian rocks or the main intrusive rocks (trachyte porphyry and quartz trachyte porphyry). The remaining three joint sets are somewhat less common and less well developed.

The Joint sets mapped at the mine are described as follows:

i) Foliation Joints

Foliation joints, which were only observed in the Precambrian rocks, generally strike northwest/southeast and dip moderately to the southwest (see Table I). These joints are well developed and are considered likely to be continuous over a number of benches.

ii) Joint Set A

Joints of Joint Set A are present in all of the three main rock units. In the Precambrian rocks, these joints strike northwest/southeast and dip relatively steeply to the northeast. They are relatively short compared to the foliation joints and are not expected to be continuous over more than about 10 feet.

In the intrusive rocks, Joints of Joints Set A dip both to the southwest and the northeast, generally at a dip of $>80^o$ (see Photo 1). These joints are very well developed and are parallel to one of the directions of regional faulting. Joints of Joint Set A are usually continuous over at least one bench.

iii) Joint Set B

As for Joint Set A, joints of Joint Set B are present in all of the three main rock units. In the Precambrian rocks, these joints strike northeast/southwest and dip steeply northwest. They are relatively short compared to the foliation joints and are anticipated to be only slightly more continuous than joints of Joint Set A in the same rock type.

In the intrusive rocks, joints of Joint Set B dip steeply to both the northwest and southeast (see Photos 1 and 2). These joints are at least as continuous and well developed as joints of Joint Set A and are also parallel to one of the directions of regional faulting.

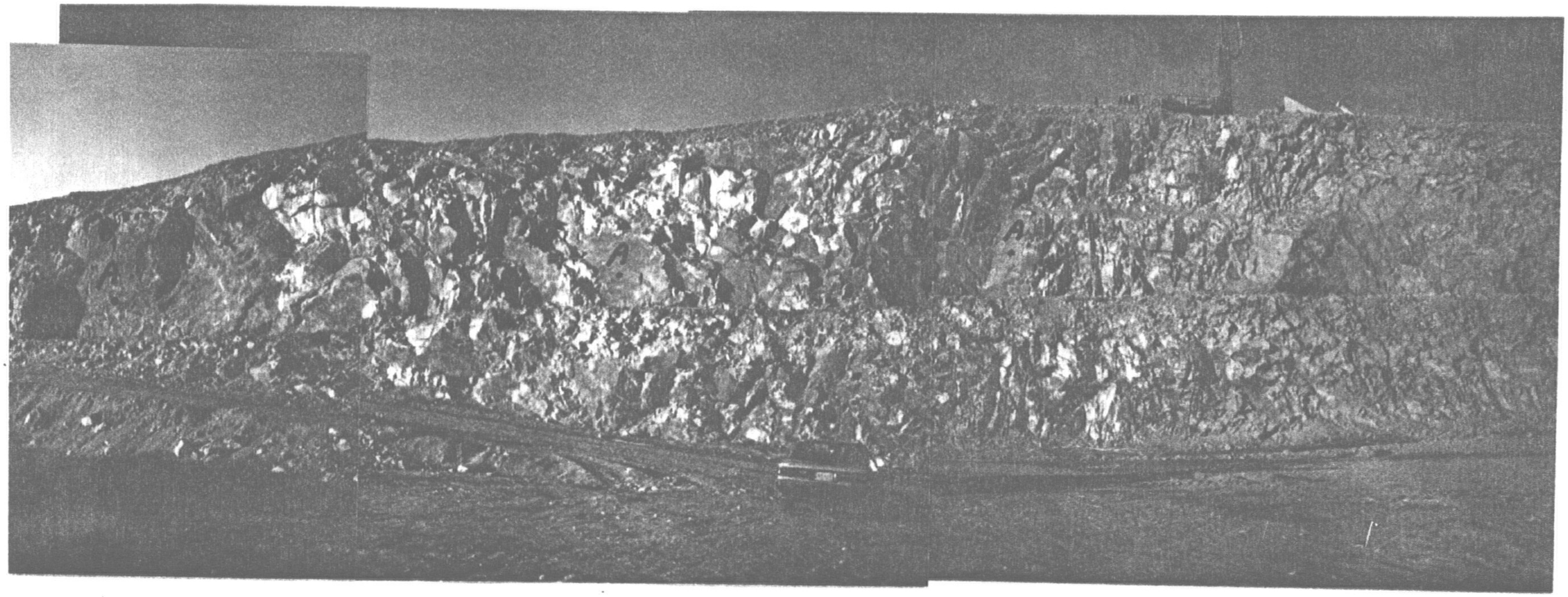

Photo 1. Joints of Joint Sets A and B in Quartz Trachyte Porphyry in the Union Hill Stock.

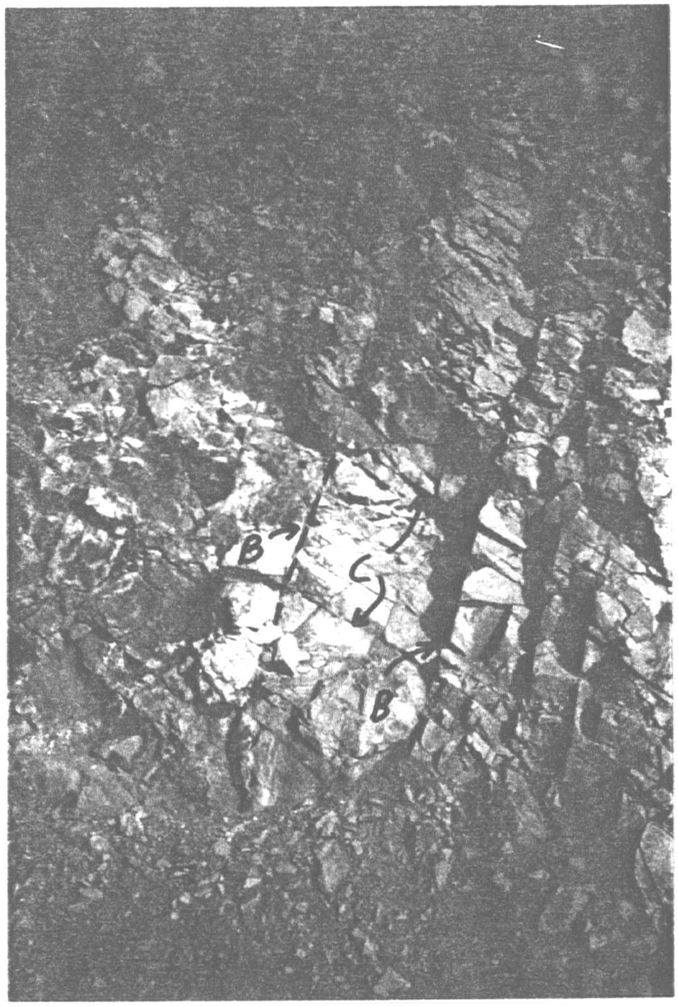

Photo 2. Joints of Joint Sets B and C in Trachyte Porphyry in the Union Hill Stock.

iv) Joint Set C

Joints of Joint Set C do not appear to be nearly as well developed or as continuous as joints of Joint Sets A and B. These joints, which were observed in only the trachyte and quartz trachyte porphyries, typically strike north to east and dip easterly to southerly at a flat to moderate dip (i.e. $<50^{\circ}$ and often about 25°). These joints are often offset or cutoff by joints of Joint Sets A or B.

v) Joint Set D

Joints of Joint Set D, which are relatively poorly developed and were observed in only the trachyte porphyry, hornblende trachyte poryphyry and Deadwood sediments, strike east/west and generally dip steeply south. Due to a lack of sufficient exposure, their continuity is unknown.

vi) Joint Set E

Joints of Joint Set E were only observed in the hornblende trachyte porphyry. These joints strike north/south and dip near vertical. Due to a lack of sufficient exposure, their continuity is unknown.

It is noteworthy that the structures discussed above and summarized in Table I are in close agreement with mapping conducted by MacLeod (1986).

3.3.2 Faults

As discussed above, there are two main regional fault sets that appear to intersect the pit area. These fault sets are essentially parallel to Joint Sets A and B. That is, the fault sets strike northwest/southeast (i.e. Set A) and northeast/southwest (i.e. Set B) and dip steeply to the northeast and northwest, respectively. While northeast/southwest striking fault zones typically range up to about 300 feet wide, the northwest/southeast trending faults tend to be discrete structures or relatively narrow fault zones. Most fault zones appear to be characterized by normal displacement and by a high degree of breakage with little or no fault gouge.

3.4 ROCK COMPETENCY

In addition to the structural geology aspects described above, a rational prediction of the likely behaviour of a rock mass also requires an assessment of the general physical and mechanical properties of the rock mass. Such parameters as susceptibility to mechanical deterioration, alteration and strength characteristics may be of key importance in evaluating rock mass behaviour.

Based on a limited number of core logs and core photographs, a brief assessment of rock quality was undertaken, the results of which are summarized in Table II. From this table it can be seen that the Rock Quality Designations (RQD) of the available core tends to improve with depth. That is, the rock tends to be less broken (i.e. more massive) with depth. It can also be seen from this table that within the trachyte porphyry the sulfide rocks are generally less broken that the oxide zone rocks. Selected photographs of drill core and core descriptions contained within drillhole logs generally confirm these findings. Drillhole logs and observation also indicate that there is a marked difference in the degree of fracturing between the trachyte porphyry and the quartz trachyte porphyry, with the quartz trachyte porphyry generally being the more massive. Based on observations in the oxide pit at the mine, it would appear that the average block size (i.e. average joint spacing) in the quartz trachyte porphyry is in the order of 3 to 5 feet. For the trachyte porphyry, the average block size is only about 1 to 2 feet. With the exception of fault zones, it is anticipated that most of the rocks that will be exposed on the pit walls will be competent, with the degree of competency increasing with depth.

3.4.1 Unconfined Compressive Strength of Intact Rock

Most rocks examined during the site visit were relatively hard and strong, having an estimated hardness of at least R4* (i.e. an estimated unconfined compressive strength of at least 8,000 to 16,000 psi). In order to confirm the estimated hardness and related unconfined compressive strengths, point load index tests were conducted on selected rock cores from the mine.

Results of point load testing of the main rock types are summarized in Table III, where it can be seen that the trachyte porphyry and quartz trachyte porphyry have estimated unconfined compressive strengths of between about 12,000 and 30,000 psi. Relatively little difference in strength is apparent between the oxide and sulfide rocks. It can also be seen that the Precambrian schist is strong, particularly perpendicular to foliation, where strengths of up to 30,000 to 40,000 psi were obtained. Even in its weakest orientation (i.e. parallel to foliation) strengths of 7,000 to 10,000 psi were obtained. Other rock types that were not tested, such as the hornblende trachyte porphyry and the Deadwood sediments are expected to be at least as strong as the trachyte porphyry and quartz trachyte porphyry.

It is noteworthy that unconfined compressive strengths of the prophyry rocks, reported by Minproc Engineers Inc in the prefeasibility study and by Dr. Z. Hladysz of the South Dakota School of Mines and Technology,

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* See Appendix B for a description of the relationship between hardness and unconfined compressive strength.

range from about 9,000 to over 14,000 psi. While the testing method for this work is unknown, it can be seen that the reported values are in general agreement with the strengths determined for this study. In any event, the rocks at the Gilt Edge Mine are all classified as ranging from hard to very hard. Although some detailed assessments of intact rock strength and deformation properties may be warranted in the future (depending on future mining plans), further testing of the intact rock is not required at this time.

3.4.2 Discontinuity Strength

Little is known concerning the shear strength of discontinuities within the rock mass. However, as most of the discontinuties that are likely to control the stability of the slope are steeply dipping (i.e. $>70^{o}$), detailed assessments of discontinuity shear strength are considered unnecessary at this time. For purposes of stability analyses, foliation joints in the Precambrian schist are assumed to have negligible cohesion and a friction angle of 30^{o}. All other joints are assumed to have negligible cohesion and a friction angle of 35^{o}. As many of the joints have a relatively high degree of roughness, the assumed discontinuity strengths are likely conservative. Depending on the behaviour of the slopes as the pit is developed, direct shear testing of joints (particularly the moderately dipping structures) may be required in the future.

3.5 HYDROGEOLOGY

A fairly extensive groundwater study has been conducted by EnecoTech Inc, of Denver, Colorado. The following discussion is based on a draft report prepared by EnecoTech (May, 1988). Much of the data presented in the EnecoTech report was obtained from the environmental baseline field study that is being conducted by Brohm Mining Corporation.

EnecoTech has identified three aquifers at the site: a shallow aquifer comprised of surficial sediments which are present primarily in the creek valleys; a fractured bedrock aquifer which underlies the entire mine area; and a sedimentary rock aquifer to the north and northeast of the mine area, which includes limestone and other sedimentary rock of paleozoic age. Based on piezometers installed in the various aquifers throughout the mine area, regional groundwater flow is determined to be from west to east.

The shallow aquifer is comprised of both alluvial and colluvial soils. The alluvial sediments which underlie the creek valleys are known to be saturated, whereas colluvial soils which mantle higher ground are expected to be unsaturated, except in local areas where perched watertables may exist. In the immediate area of the proposed pit, the shallow aquifer is only expected to be saturated on the western side of the pit, where tributaries of Strawberry Creek will be truncated by the west wall of the pit. To reduce seepage into the pit and groundwater recharge to lower slopes, some form of shallow groundwater control, such as interception ditches located on upper benches, will be required.

The fractured bedrock aquifer which underlies the proposed pit is currently saturated to an elevation of about 5250 ft. The ultimate pit will therefore penetrate about 700 to 1000 feet below the present watertable. Due to the fracturing present in the rock mass, the pit walls are expected to drain as the pit is developed and, with the exception of some local areas where perched water tables may be present, the phreatic surface should be located near the base of the pit. Provided fractures daylight in the bench faces, individual benches are also expected to be well drained. Artificial drainage measures, such as drainholes, are not likely to be necessary.

4. SLOPE STABILITY ANALYSES AND SLOPE DESIGN

Slope stability analyses involved investigating all kinematically possible failure modes (i.e. failure modes involving discrete blocks, formed by discontinuities, which are free to slide or topple into the excavation) which could lead to shallow failure of individual benches, and/or deep seated failure involving large sections of the overall slope. Consideration was also given to the limited nature of the structural data that were available; to the performance of the existing initial slopes with respect to evaluation of basic failure modes and general rock mass behaviour; and to the phased development plan for the pit, which will see a number of interim pits excavated before ultimate pit slopes are exposed.

4.1 BASIC SLOPE DESIGN CONSIDERATIONS

In rock slopes, instability may result from failure along structural discontinuities such as bedding, joints, geological contacts, faults, etc. (i.e. kinematic failures). In high slopes or slopes in relatively weak rock or altered rock, instability may also develop as a result of failure through intact rock or along a deep seated failure surface consisting of a combination of discontinuities and intact rock. In analyses of deep seated failure, an assessment of the rock mass strength is usually required.

When assessing failure mechanisms related to structural discontinuities, the most important factors are the orientation, geometry and spatial distribution of discontinuities in the slope. It is also important to evaluate these discontinuities with respect to both the orientation and alternative possible angles of the proposed pit slope. It is following these basic principles that the slope stability analyses were carried out.

Slope control can be basically accomplished in two ways:

a) to design the slope so that no failures occur or

b) to excavate the pit under controlled conditions and to design the slope with adequate access so that failures can be caught on berms and accordingly removed, if necessary.

The first solution is usually too conservative to be economically feasible. The second solution requires thorough consideration of slope geometry so that failures are contained on berms and safe access is available to the berms to allow removal of collected debris, if required. This solution provides adequate safety at minimal cost, although special design or remedial measures may be required to ensure the stability of haulroads or critical installations on benches.

The parameters which govern the geometry of a slope are shown in Fig. 3. These parameters are primarily controlled by the strength and nature of the rock. Bench height should be selected to provide a safe working slope as well as an optimum interramp slope angle. It should be noted that, without affecting the slope angle, higher benches will allow wider berms for better protection and more reliable and easier access, if this is desirable, although the size of possible failures may increase.

Berm width should be controlled by the access required to the slope, as well as by the optimum width to accomodate failures. It must be accepted that, even with careful perimeter blasting techniques, some breakback may occur. In general, slopes should have berms wide enough to trap falling debris and, if and where desired, provide sufficient access for equipment to keep berms clean and effective as catchments.

By inclining the bench faces, blasting damage is reduced and high stresses are less likely to develop near the bench crests. Hence, tension cracks and overhangs are minimized. Avoiding these problems accordingly reduces the amount of rockfall and increases the safety of the slope.

4.2 ENGINEERING GEOLOGY AND DESIGN SECTORS ON THE PIT SLOPES

Rational slope stability analysis and slope design requires prediction of the geologic structural conditions which will occur on the pit walls. Such a prediction includes determining the distribution and location of lithologic units, major structures, etc. (i.e. Structural Area boundaries). At Gilt Edge, the basic approach to preliminary slope design was to consider slopes in the various structural areas separately.

Not only must structural areas be considered for individual analysis; the overall orientation of the pit walls must also be considered. Different pit wall orientations may require different design considerations. Hence, it is usually necessary to define zones which contain one structural area and one general slope orientation. These zones are designated design sectors.

All available geological information was used to project the structural area boundaries onto the proposed pit walls. The structural area and straight slope segments on the proposed walls were used to determine the design sector boundaries and delineate the design sectors shown in Figs. 4, 5, and 6. For analysis purposes, and because it is understood that the ultimate pit limits shown in Fig. 6 are preliminary in nature and may change appreciably, a full range of possible design sectors was defined. That is, rather than just evaluating the design sectors illustrated in the two interim pits illustrated in Figs. 4 and 5 and the ultimate pit illustrated in Fig. 6, it was assumed that each of the main rock units (i.e. the precambrian schist, trachyte porphyry and quartz trachyte porphyry) could have any slope orientation. Thus, a full suite of design sectors were defined at 30° increments of slope orientation (i.e. 000°, 030°, 060°, ... 330°). Alternative slope design criteria were then established for each possible design sector and the appropriate results applied to the potential design sectors illustrated in Figs. 4, 5 and 6. Information relating to pit wall orientations and related slope information for the full range of possible design sectors is given in Tables IV, V, and VI.

4.3 ASSESSMENT OF POSSIBLE DEEP SEATED FAILURE MECHANISMS

A brief assessment was carried out to assess the possibility of deep-seated failure, involving several benches or even the whole slope. Failures of this nature may involve major throughgoing faults or other major discontinuities. In this regard, both individual major structures and/or the average orientation of fault sets should be considered.

At the Gilt Edge Mine the only major structures along which deep-seated failure appears to be able to occur are the two dominant regional fault systems (i.e. the faults that are parallel to Joint Sets A and B). However, as these faults appear to dip very steeply (i.e. $>75^{\circ}$) with respect to the possible overall slope angles, the possibility of such a failure mechanism occurring would seem to be remote.

Deep-seated failures may also develop through a complex interaction of minor and/or major structures to form a large continuous failure surface. In some cases, deep-seated failure planes can, in part, develop by failure through intact rock bridges between discontinuities. Such failures are often precipitated by small slope readjustments and high stress concentrations at the toe of the slope, which tend to exceed the strength of the rock mass. This type of failure mechanism is particularly common in slopes where softer material at the toe of slope, such as coal, potash or schist bands, is squeezed out due to the load which is applied by the overlying strata.

For engineering purposes, the principal consideration in evaluating the compressive strength of the rock mass is to determine if there is any possibility of excessive deformation, and ultimately, rotational failure of the slope.

This would be due to failure of the intact material, resulting from the weight of the overlying strata.

Based on the relatively high hardness and high compressive strength of most intact rocks in the pit, the likelihood of time dependent strain and subsequent failure of the rock mass appears to be small. Hence, the possibility of deep-seated rotational failure due to failure of the intact material involving the whole slope is low (at least within the range of overall slope angles which are geometrically feasible, and which are considered in this analysis).

4.4 STABILITY ANALYSES AND ASSESSMENTS OF KINEMATICALLY POSSIBLE FAILURES

4.4.1 Slope Design Based on Orientation of Geological Structure

As discussed above, individual discontinuities or combinations of discontinuities may form discrete blocks which could result in failure of the slope or benches (i.e. kinematically possible failure modes). Because there are variations in intensity, orientation and dip of the joint and fault sets throughout the mine, the significance of one particular failure mode on the stability of a particular slope may be low. However, the combined significance of all possible failure modes, with respect to both the actual and predicted performance of the slope may be high and should be evaluated to prepare a rational slope design.

Over the exposures mapped at the mine, many structures are continuous over at least 20 to 30 feet. This is particularly true of foliation joints in the Precambrian schist and of Joint Sets A and B in the intrusive rocks. However, because of the generally steep nature of most structures in the mine area, particularly the longer structures, it is unlikely that failure would develop over more than one bench height.

4.4.2 Determination of Kinematically Possible Modes of Failures

Lower hemisphere equal area projections of planes representing the peak orientations of the various fault sets and joint sets in each possible design sector were used to define possible failure modes. Failure modes which are considered to be kinematically possible are illustrated on the equal area projections in Appendix C.

4.4.3 Mechanical Stability Analyses

Simple limit equilibrium stability analyses were carried out for each possible failure mode using computer techniques. As discussed in Section 3.4.2, negligible cohesion and a friction angle of 35^{o} were assumed for most discontinuities, with only foliation joints having a lower assumed shear strength of zero cohesion and a friction angle of 30^{o}.

To simplify the analysis, drained slopes were assumed, and a Factor of Safety greater than or equal to 1.2 was considered adequate for stability of dry (i.e. drained or dewatered) slopes. A Factor of Safety greater than or equal to 2.0 (assuming the dry condition) was considered adequate for stability of slopes subject to adverse groundwater conditions (i.e. undrained or fully saturated slopes).

4.4.4 Assessment of Possible Failures

In most areas, with the exception of a few design sectors where no significant failure modes are apparent, the principal kinematic controls are wedge and planar failures on benches. As shown in Tables IV, V and VI, these potential failures vary in terms of their intensity or importance. That is, the relative degree of development of individual discontinuity sets or combinations of discontinuity sets which form potential failures, and therefore the likelihood of bench faces consistently breaking back to or forming along such failures, is variable. Based on the estimated intensity/importance of the principal kinematic controls summarized in Tables IV, V and VI, and on engineering judgement, the apparent plunge or dip of potential failure modes considered to control stability of individual benches was selected (see Tables IV, V and VI).

For example, in Design Sector TP-180 (see Table V), the numerous strongly developed planes at an apparent dip of 81° are felt to control the stability of the benches. While the potential for planar failures on Joints of Joint Set C which dip at about 53° is recognized, this joint set is not felt to be strongly enough developed to control the design of the bench. In this case, breakback along joints of Joint Set C is only considered to be a local problem, unless overblasting results in opening and/or extension of these joints.

In general, because most of the strongly developed kinematic controls are steeply dipping, it is unlikely that large deep seated failures could develop in the pit. Hence, in terms of design of interramp slopes, bench geometry will control the overall slope design in most areas.

4.4.5 Bench Breakback Analysis

A brief study of bench breakback was carried out to evaluate the behaviour of existing benches at the mine. Bench breakback was obtained by measuring directly the bench face angles of the few existing benches at the mine. Results of this brief assessment indicate that, as expected the quartz trachyte porphyry benches stand the steepest, with the bench face angles ranging from 70° to 80° for a 40 foot high bench. Bench face angles of 60° to 65° were observed in trachyte porphyry, with bench face angles in trachyte breccia being about 65° to 70°. Berms are relatively clean and free of rockfalls and ravelling debris, and range from about 15 to 20 feet wide.

4.5 ALTERNATIVE INTERRAMP SLOPE DESIGNS

As the possibility of deep seated failure involving multiple benches on large portions of the slope is low, slope design has been carried out by assessing possible bench failures and related alternative bench geometries for the main rock units (i.e. Precambrian rock, trachyte porphyry and quartz trachyte porphyry) that will be exposed over the pit slopes. These assessments can then be used to select optimum bench geometries; and hence, to develop a preliminary optimum design of overall slopes. Consideration can also be given to the results of bench breakback, groundwater and other relevant aspects.

The interim and ultimate pit configurations illustrated in Figs. 4, 5 and 6 were developed by Brohm Mining. The interim pit configurations represent uniform pit wall slopes of 50°, whereas the ultimate pit configuration represents a uniform overall slope angle of 54° at an average overall slope height of about 1200 feet. At the time of the site visit, 40 foot high double benches, comprised of two 20 foot high single benches were being excavated. Alternative preliminary slope designs based on the results of the above work are summarized in the following and in Tables IV, V and VI.

4.5.1 Bench Geometry

i) Bench Height

Based on assessment of the engineering geology and the kinematic controls, and because of the advantages of achieving bench heights as high as possible (with due consideration of safety and efficiency), 40, 60, 80 and 100 foot high benches were evaluated in all design sectors.

ii) Bench Face Angle

Design bench face angles are assumed to be 90°. In most design sectors, it is anticipated that with careful controlled excavation, it should be possible to achieve effective bench face angles of 70° to 80°. Generally, effective bench face angles after breakback have been equated to the apparent plunge or dip of the principal kinematic control. However, in the Precambrian rocks, breakback is usually assumed to be only half of that indicated by the apparent plunge or dip of the principal kinematic control. A maximum bench face angle of 80° is assumed for all benches. Although some small failures will no doubt occur with these steep bench face angles, large scale consistent breakback of final wall benches is not anticipated if appropriate perimeter excavation techniques are successfully employed.

iii) Berm Width

As shown in Fig. 3, total berm width is defined as being the sum of the breakback of the bench crest (i.e. between vertical and the effective bench face angle) and the effective berm width required to provide access, contain small failures and rockfalls, etc. At the Gilt Edge Mine, effective berm widths of 25, 28, 32 and 35 feet are recommended for 40, 60, 80, and 100 foot high benches, respectively. These berm widths provide greater catchment width than is provided in the presently excavated benches, and allow for some additional breakback of the benches without the berms becoming so narrow as to be ineffective.

4.5.2 Interramp Slope Angles

Based on the assessments carried out and the bench geometries discussed above, the range of preliminary interramp slope angles which appears feasible in the possible design sectors is summarized on the right hand side of Tables IV, V and VI. As can be seen, significantly steeper interramp slope angles can be achieved if the bench height is increased. However, increased care and caution with drilling, blasting and excavating must also be taken when increasing the bench height. As will be discussed below, improved excavating procedures will have to be developed with trial slopes to ensure that steeper and higher slopes can be safely mined.

In calculating the steeper interramp slope angles for the higher benches, it has been assumed that the effective bench face angles remain fixed as the bench height is increased. While this assumption may be valid from a geotechnical standpoint, it is often very difficult to accomplish from an operational standpoint. Flatter bench face angles may result due to blasting problems, overdigging, etc. It is particularly difficult to maintain very steep bench face angles when a number of benches are stacked together.

The consequences of not maintaining bench face angles as steep as possible can be significant. As shown in Table VII (a), if effective berm widths of 25, 28, 32 and 35 feet are maintained for 40, 60, 80 and 100 foot high benches, respectively, the effect of flattening the bench faces from 80° to 65°, would be to flatten the interrramp slope angle by as much as about 9° to 11°. Such decreases in slope angle could have a considerable economic impact on mining. Table VII (b) illustrates the effect on initial berm widths of breakback of the benches and associated flattening of the bench face angles. If berms with an initial 80° bench face angle and initial berm widths of 25, 28, 32, and 35 feet (for 40, 60, 80 and 100 foot high benches, respectively) breakback to bench face angle of as low as 65°, a significant loss in catchment (i.e. berm width) will result. In this regard, berm widths of less than 12 feet could result if bench faces break back to 65°. Berm widths of less than about 20 feet are considered to be inappropriate, particularly for bench heights of greater than about 60 feet.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 PRELIMINARY SLOPE DESIGNS AND TRIAL SLOPES

Based on the results of the slope stability analyses summarized in Section 4 and on experience, it is concluded that where the structure will allow steep bench face angles (such as in nearly all design sectors in the quartz trachyte porphyry), it should be possible to excavate 80° bench face angles for benches up to 60 feet high. For higher benches, bench face angles of about 70° to 75° are probably more realistic.

For preliminary slope design purposes, it is recommended that 60 foot high benches be utilized. Other bench configurations could be incorporated on a trial basis during early phases of mining before establishment of ultimate slopes begins. To illustrate the application of 60 foot high benches on the proposed interim and ultimate pit configuration in Figs. 4, 5 and 6, the design sectors illustrated on the pit configurations were matched with the applicable alternative slope configurations summarized on Tables IV, V and VI. The appropriate recommended interramp slope angle was then determined and noted on the pit configurations. From the circled interramp slope angles on Figs. 4, 5 and 6, it can be seen that:

- interramp slope angles in quartz trachyte porphyry will be about 56° to 57°.
- interramp slope angles in trachyte porphyry will range from about 50° to 57°, depending on wall orientation.
- interramp slope angles in Precambrian rocks will generally be about 57° for most wall orientations. The most adverse wall orientations in these rocks are those which will be cut approximately parallel to the strike of foliation. In these cases, interramp slope angles in the 43° to 45° range may be necessary. Such adverse wall orientations

are likely to only exist over a small portion of the east wall of the ultimate pit. However, this should be investigated further during development of the pit.

It is noteworthy that the incorporation of ramps onto the pit slopes will result in a slope with a shallower overall slope angle than the interramp angles discussed above.

While detailed analyses and determination of alternative slope designs have not been carried out for the Deadwood sediments (due to their relative insignificance in terms of the slope configurations and due to a lack of data), they may be exposed at the crest of the north wall of the proposed ultimate pit over a slope height of up to 100 to 200 feet. As kinematics are generally favourable in this rock unit, it is recommended that an interramp slope angle of 54^{o} (comprised of 60 foot high benches with 75^{o} effective bench angles and 28 foot wide berms) be used for preliminary design.

The slope designs discussed above are intended to apply to the sulfide rocks. While the near surface oxide rocks are expected to be similar to the sulfide rocks from a structural standpoint, the oxide and near surface weathered rocks are bound to be somewhat less competent. Thus, it is expected that it will be more difficult to hold steep bench faces in these zones. A somewhat greater number of rockfalls and increased ravelling and breakback of bench crests can be expected.

Because of the nature of open pits and geological conditions, theoretical final slopes often include adjacent design sectors with different designs, and/or portions of design sectors that could theoretically be split off as separate small design sectors. Zones of transition between adjacent design sectors, or small portions within a larger design sector, have not been included in the above slope designs. In general, recommended slope angles should not be exceeded in these zones. However, because of the amount of berm width provided in the slope designs (i.e. to account for bench breakback) it is considered reasonable in such areas to allow slight deviations (i.e. within about 2^o) in interramp slope angles from those recommended. While it is recognized that steepening could result in additional breakback and narrower bench widths, increasing the possibility of rockfalls, steepening or flattening the interramp slope in small areas would allow more rational incorporation and blending of the recommended slope designs into the overall pit plan.

While slope designs incorporating 80 and 100 foot high benches are felt to be possible from a kinematic and slope stability standpoint, there may be some operational or other constraints that will preclude the use of 80 and 100 foot high benches. In addition, and as discussed above, it will no doubt be more difficult and require more care with all phases of the excavation process to develop 80 and 100 foot high benches, as opposed to 60 foot high benches. Thus, to fully assess the preliminary slope designs, it is recommended that trial slopes be developed before final walls are excavated. These slopes should incorporate such practices as trial perimeter blasts to evaluate all aspects for potential wall steepening. It is noteworthy that the phased approach to mining the Gilt Edge Pit (i.e. a series of pushbacks before the final wall is excavated) should allow ample time for trial slopes to be evaluated.

Trial slopes should be developed in areas where failures can be allowed to occur without affecting the efficiency and safety of the operation. If failures or other problems do occur on a particular trial section, provision should be made so that the failure can be controlled and the mining geometry modified without jeopardizing the safety of the mine.

Based on the results of trial slopes, ongoing geotechnical assessments and mine planning studies, updated slope designs can be prepared on an as needed basis.

5.2 SLOPE GEOMETRY AND EXCAVATION TECHNIQUES

The hazard of local instability, rockfalls and general ravelling increases as both the jointing and blasting damage increases. Other than thorough scaling and possibly local application of mesh and rock bolts, little can be done to control unfavourable effects of jointing. However, blasting damage can be minimized by control perimeter blasting on the final wall and careful production blasting.

5.2.1 Control Blasting

Control blasting is recommended in all areas of the interim and final pit walls to maintain rock mass strength within the slope and to control excessive bench breakback. Besides increasing the stability of the slope, certain types of control blasting on benches might allow larger charges to be used in production blasts. This could lead to increased efficiency and better flexibility to achieve the required fragmentation.

The type of control blasting technique to be used on the slopes depends on the results of blasting trials. Effects of using small diameter, closely spaced, inclined preshear holes, cushion blasting or post splitting, unstemmed and stemmed large diameter holes with cardboard cartridges, buffer blasting, sequential blasting techniques in both cushion and buffer blasts as well as in the production blasts, etc. could be considered in blasting trials on interim slopes, as required.

To protect haulroads, regardless of location, it is recommended that a more sophisticated or higher level of control blasting be carried out on the benches immediately above and below the haulroad. If contol blasting is not sufficient to maintain the integrity of the slopes, berms may be reinforced by simple artificial support in site specific areas.

5.2.2 Production Blasting

Production blasting should be designed so that blasting damage to the pit wall is minimized. With regard to benches, for example, this can be achieved to some extent by reducing subgrade drilling and having blastholes span the bench crests.

Accurate location of all blastholes by surveying is essential. Optimum results can only be obtained by varying blasting techniques and correctly supervising and designing field trials where loads, spacing, burden, delays, etc. are varied to obtain the best possible results. It is sound engineering to make special provision for a series of test blasts to ensure optimum results for existing operating conditions.

5.3 SURFACE WATER AND GROUNDWATER CONTROL

An efficient system of surface drainage ditches should be maintained to control surface water behind and in the pit, with the surface water directed away from the pit. Ditches on haulroads and specific benches would also help to control runoff from both precipitation and snowmelt.

As discussed in Section 3.5, the pit walls are expected to drain as the pit is developed and, with the exception of some local areas where perched water tables may be present, the phreatic surface should be located near the base of the pit. Provided fractures daylight in the bench faces, individual benches are expected to be well drained.

5.4 CLEANING BERMS AND SCALING

All benches should be adequately scaled to minimize rockfalls. This is particularly important if 80 or 100 foot high benches are utilized. Debris buildups may require cleanup at a later date in certain areas. If possible, berms should be kept relatively free of excessive buildup of rockfalls and ravelling material to maintain adequate catchments. Ideally, if berms are accessible from both ends, access will not be lost if a bench failure occurs.

5.5 MONITORING SLOPES FOR MOVEMENT

Performance of all slopes should be carefully evaluated on an ongoing basis by regular visual field surveys and/or other forms of monitoring systems to detect slope displacements. Movement of the top, as well as the bottom, of slopes should be recorded. It may be feasible to install a straight line of survey hubs at a few key locations on the slopes. Direct distance measuring equipment can be used to considerable advantage in this work as well.

Because of the difficulty of making absolute predictions about the behaviour of the slope, there is always the possibility that the most carefully engineered slope will become unstable and will require remedial measures. Thus, the purpose of monitoring would be to provide data about the current and anticipated stability of the slope. Also, for purposes of efficiency and safety, this data would assist in making rational design modifications as mining proceeds.

5.6 SUMMARY OF RECOMMENDATIONS FOR FURTHER GEOTECHNICAL WORK

A number of additional tasks which should be carried out by mine personnel to further evaluate and update the preliminary slope design studies summarized herein are outlined in the following. Piteau Associates would be pleased to assist Brohm with planning and initiating this work.

i) Geologic Structural Mapping

Because of the importance of the geological structure to slope design, and the very limited data base which is presently available, detailed geologic structural mapping should be carried out on an ongoing basis by mine personnel as mining proceeds. Mapping of major faults and/or fault sets and joint sets should be carried out to update the geologic structural mapping, to determine if the structural populations are changing, and thus to determine if any slope design modifications are required.

ii) Core Logging and Strength Testing

It is recommended that all diamond drill core from any future exploration programs be carefully logged. Core from the diamond drillholes should be drilled and handled with care and geotechnically logged immediately after being drilled and before being split. A limited number of core samples could be taken for further strength and direct shear testing, as required.

iii) Hydrogeological Investigations

Piezometers should be installed around the pit crest so that water levels can be monitored during mining. This monitoring data could be very useful in ongoing stability assessments conducted as the pit is developed. While the existing piezometer installations provide for adequate monitoring on a broad scale, additional installations should be installed near the pit crests. A minimum of three multiple piezometer installations is recommended. Two of these installations should be located behind the south and west walls (where Strawberry Creek is located), and the third should be located behind the northeast wall (towards the sedimentary aquifer). Existing piezometers should be falling head tested to provide data on the permeability of the rock mass on a smaller scale than that provided by the four pumping tests which were conducted by EnecoTech. Smaller scale test results would be more relevent to the drainage characteristics of individual benches than the large scale test results which are currently available.

iv) Documentation and Back Analysis of Failures

Documentation and back analysis of any slope failures which might occur should be carried out to obtain additional information on the strength of the discontinuities or rock mass. A periodic visual inspection of the pit should be carried out to determine visible signs of movement.

6. ACKNOWLEDGEMENTS

The author acknowledges with thanks the assistance and cooperation of mine personnel during the course of this work. Particular thanks are expressed to Messrs. T. Fox, R. MacLeod and J. Barron for their assistance during the field portion of the study.

Respectively submitted,

PITEAU ASSOCIATES ENGINEERING LTD.

Alan F. Stewart, P.Eng.

PROFESSIONAL
PROVINCE OF
ALAN F. STEWART
BRITISH COLUMBIA
ENGINEER

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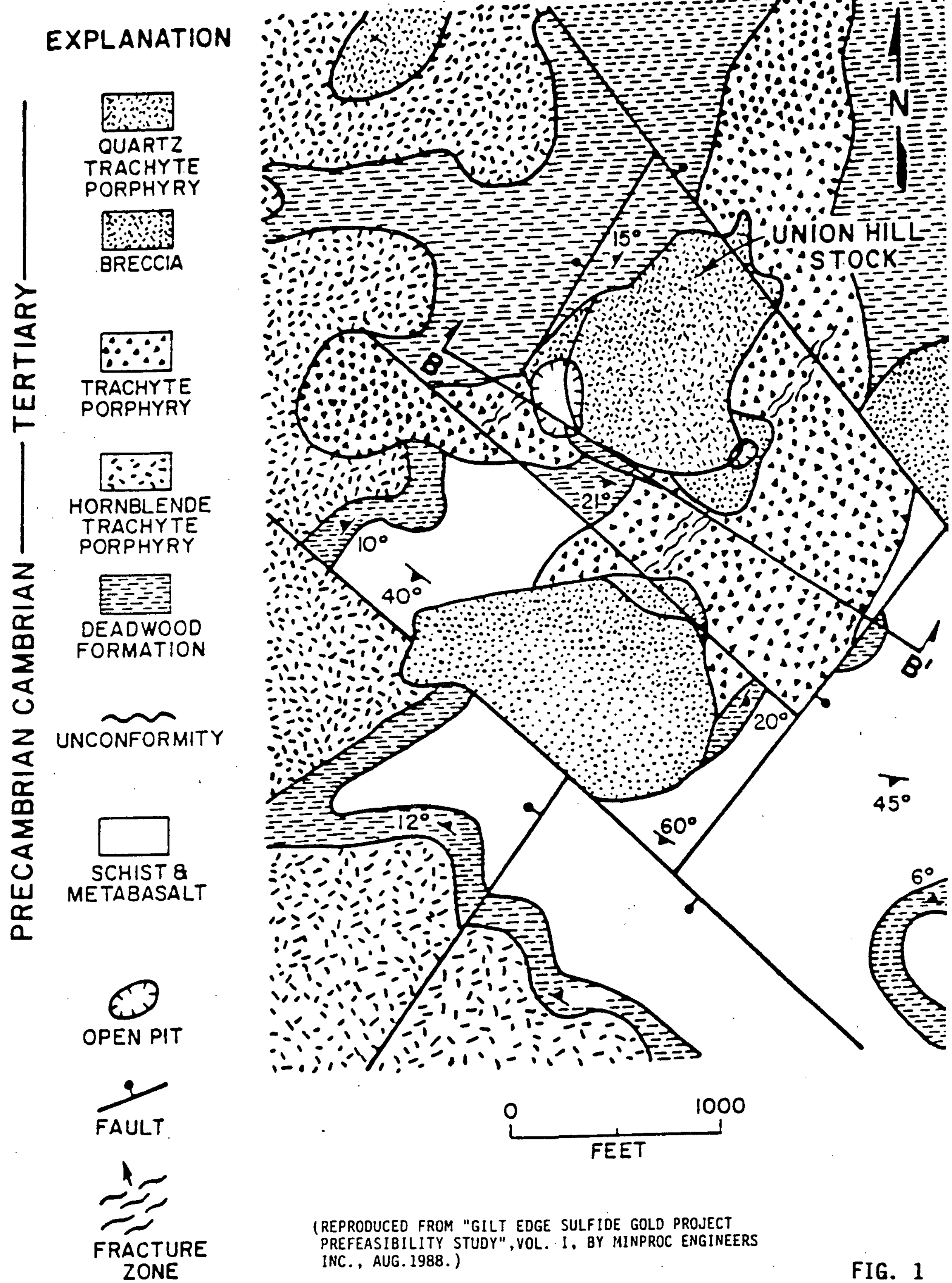
FIGURES

JOB NUMBER 89-098

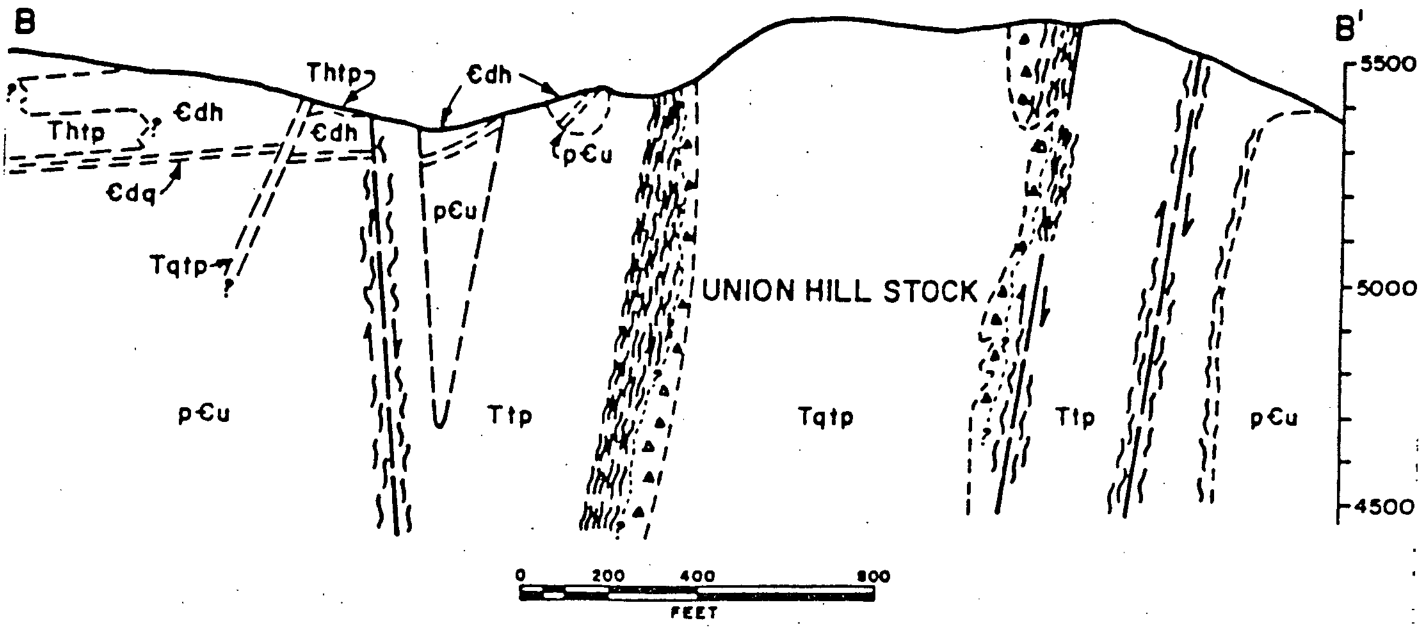


(REPRODUCED FROM "GILT EDGE SULFIDE GOLD PROJECT PREFEASIBILITY STUDY", VOL. I, BY MINPROC ENGINEERS INC., AUG. 1988.)

FIG. 1

BROHM MINING CORP. GILT EDGE MINE	PITEAU ASSOCIATES GEOTECHNICAL CONSULTANTS VANCOUVER CALGARY	
GEOLOGY OF THE GILT EDGE PROJECT AREA	BY: D.H. APPROVED:	DATE: DEC/88 DWG:

JOB NUMBER 88-098



Tqtp	QUARTZ TRACHYTE PORPHYRY
Ttp	TRACHYTE PORPHYRY
Thtp	HORNBLENDE TRACHYTE PORPHYRY
Єd	DEADWOOD FORMATION h HORNFELTS q QUARTZITE
pЄu	UNDIFFERENTIATED METAMORPHIC ROCKS
	FRACTURE ZONE / BRECCIA

(REPRODUCED FROM "GILT EDGE SULFIDE GOLD PROJECT PREFEASIBILITY STUDY", VOL. I, BY MINPROC ENGINEERS INC., AUG. 1988.)

FIG. 2

BROHM MINING CORP.

GILT EDGE MINE

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

GEOLOGICAL CROSS SECTION B-B' THROUGH THE GILT EDGE PROSPECT AREA LOOKING NORTHEAST

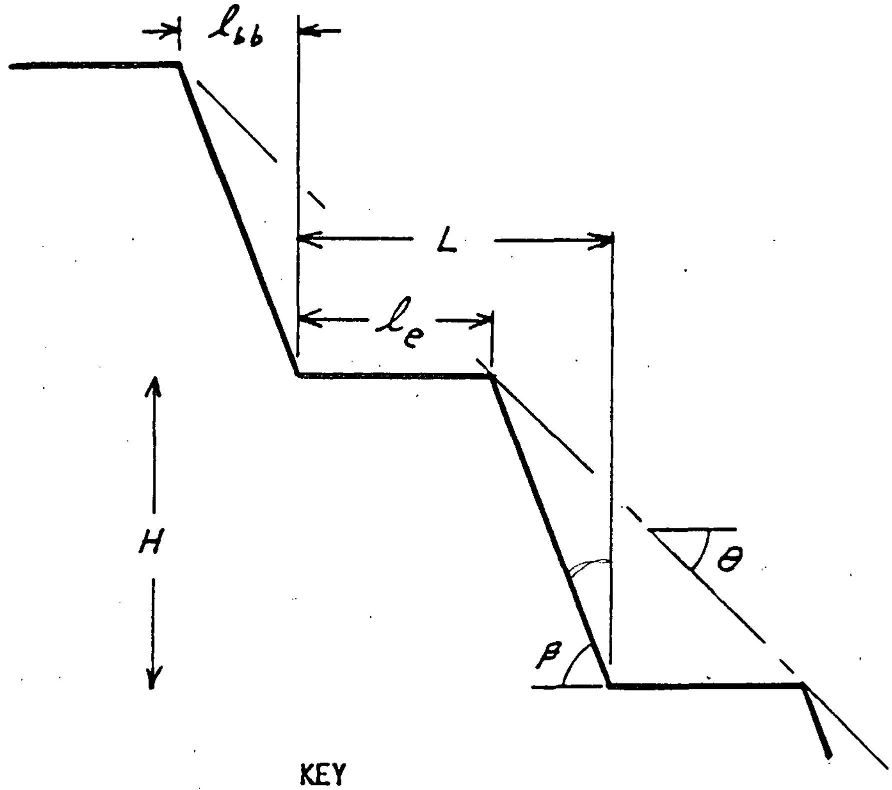
BY: DH

DATE: DEC/88

APPROVED:

DWG:

JOB NUMBER 86-198

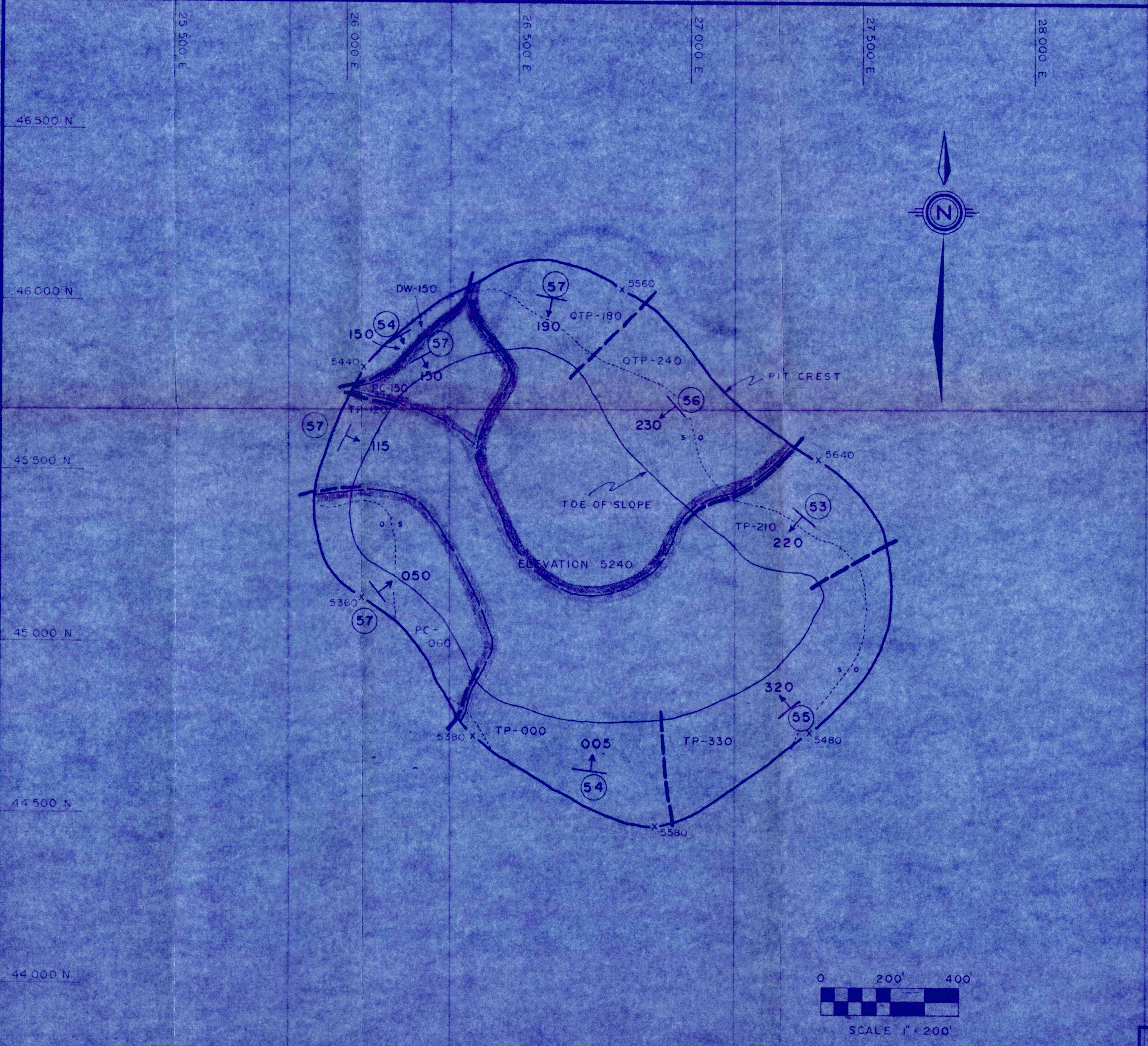


KEY

H = Bench Height
l_{bb} = Breakback of Bench Crest
l_e = Effective Berm Width
L = Total Berm Width (l_e + l_{bb})
β = Bench Face Angle (AFTER BREAKBACK)
θ = Interramp Slope Angle (INTERMEDIATE SLOPE ANGLE BETWEEN RAMPS)

FIG. 3

BROHM MINING CORP. GILT EDGE MINE	PITEAU ASSOCIATES GEOTECHNICAL CONSULTANTS VANCOUVER CALGARY
BENCH GEOMETRY PARAMETERS	BY: AFS DATE: DEC/88 APPROVED: A.S. DWG:



LEGEND

PRECAMBRIAN ROCKS

GENERALLY COMPRISED OF QUARTZ-MICA SCHIST, AMPHIBOLITE AND MINOR BANDED METACHERT

UPPER CAMBRIAN ROCKS

DEADWOOD FORMATION - INDURATED QUARTZITE AND HORNFELSED, INTERBEDDED SANDSTONE, SHALE AND CARBONATES.

TERTIARY ROCKS

TRACHYTE PORPHYRY - CRYPTOPERTHITE AND MINOR PLAGIOCLASE PHENOCRYSTS SET IN A MICROCRYSTALLINE GROUNDMASS OF MOSTLY POTASSIUM FELDSPAR

QUARTZ TRACHYTE PORPHYRY - SANIDINE, CRYPTOPERTHITE, PLAGIOCLASE AND QUARTZ PHENOCRYSTS SET IN A CRYPTOCRYSTALLINE GROUNDMASS OF LARGELY POTASSIUM FELDSPAR

SYMBOLS

5420 x SPOT ELEVATION

OXIDE/SULFIDE CONTACT

LITHOLOGIC CONTACT

DESIGN SECTOR BOUNDARY

QTP-180 DESIGN SECTOR NUMBER

230 APPROXIMATE TREND AND DIP DIRECTION OF PROPOSED PIT WALL.

54 RECOMMENDED PRELIMINARY SLOPE ANGLE BETWEEN HAULROADS

NOTE: BASED ON END OF YEAR 1 PLAN PROVIDED BY BROHM MINING CORP.

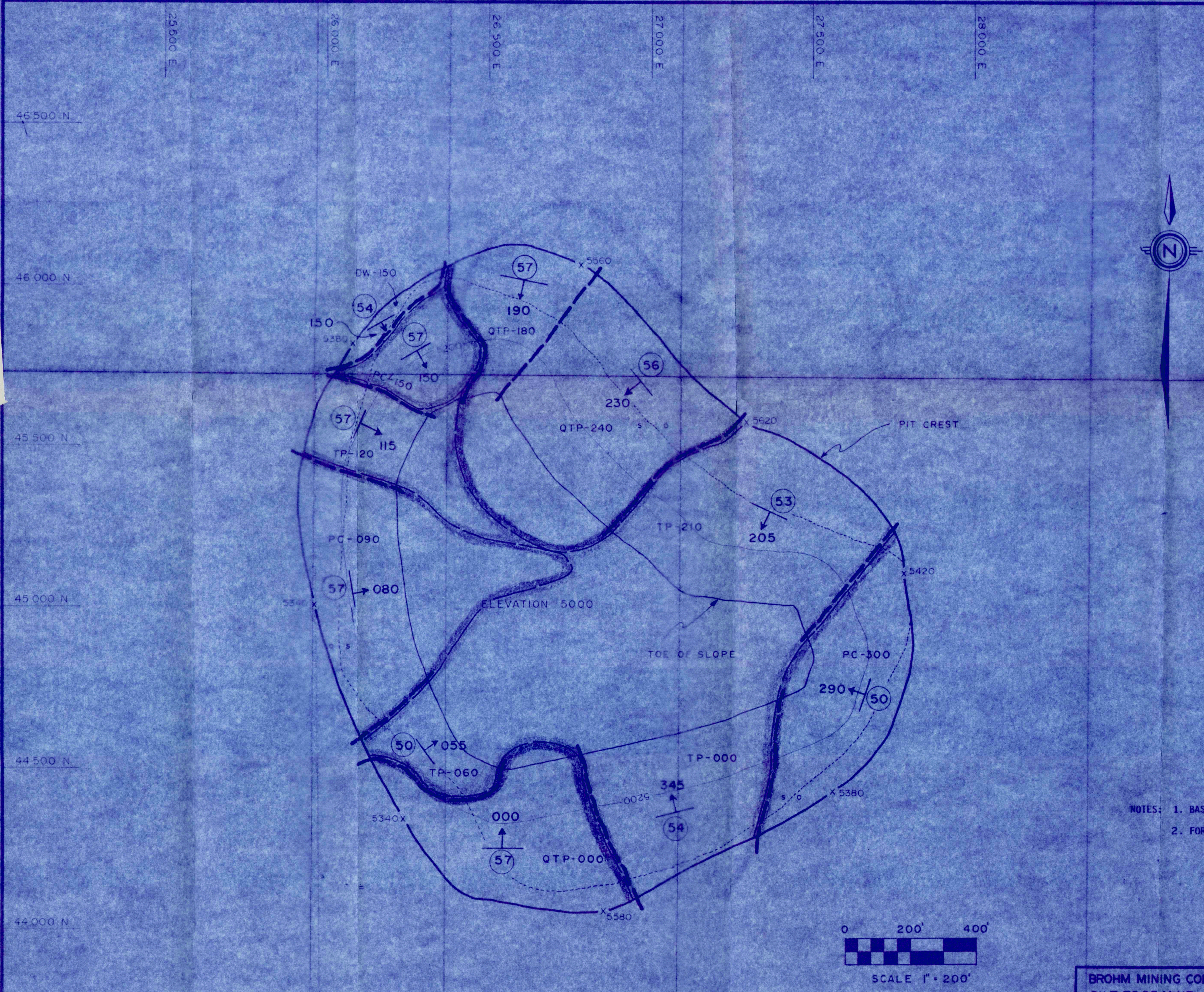
FIG. 4

BROHM MINING CORP.
GILT EDGE MINE

PITEAU ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

RECOMMENDED PRELIMINARY SLOPE ANGLES FOR INTERRAMP SLOPES BETWEEN HAULROADS FOR THE END OF YEAR I INTERIM PIT

D.H. DEC/88



NOTES: 1. BASED ON END OF YEAR 3 PLAN PROVIDED BY BROHM MINING CORP.
2. FOR LEGEND AND SYMBOLS SEE FIG. 4

FIG. 5

BROHM MINING CORP. GILT EDGE MINE	PITEAU ASSOCIATES GEOTECHNICAL CONSULTANTS VANCOUVER CALGARY		
RECOMMENDED PRELIMINARY SLOPE ANGLES FOR INTERRAMP SLOPES BETWEEN HAULROADS FOR THE END OF YEAR 3 INTERIM PIT	BY D.H.	DATE DEC/88	

NOTES: 1. BASED ON ULTIMATE PIT PLAN PROVIDED BY BROHM MINING CORP.
2. FOR LEGEND AND SYMBOLS SEE FIG. 4

FIG. 6

BROHM MINING CORP. GILT EDGE MINE	PITEAU ASSOCIATES GEOTECHNICAL CONSULTANTS VANCOUVER CALGARY	
RECOMMENDED PRELIMINARY SLOPE ANGLES FOR INTERRAMP SLOPES BETWEEN HAULROADS FOR THE ULTIMATE PIT	D.H.	DEC/88

TABLES

TABLE I

ORIENTATION OF JOINT SETS WITHIN STRUCTURAL AREAS BASED ON SURFACE MAPPING

STRUCTURAL AREA	ROCK TYPE	POPULATION	FOLIATION JOINTS				JOINT SET A				JOINT SET B			JOINT SET C				JOINT SET D				JOINT SET E			
			SET	DIP DIRECTION	DIP	%/NO.[2]	SET	DIP DIRECTION	DIP	%/NO.[2]	DIP DIRECTION	DIP	%/NO.[2]	SET	DIP DIRECTION	DIP	%/NO.[2]	SET	DIP DIRECTION	DIP	%/NO.[2]	SET	DIP DIRECTION	DIP	%/NO.[2]
1	Precambrian Schist	18	FN	213	44	17/3	A1	023	72	17/3	323	75	11/2	-	-	-	-	-	-	-	-	-	-	-	-
2	Trachyte Porphyry	159	-	-	-	-	A1 A2 A3	208 034 059	75 83 87	11/17 4/6 3/5	303 326	81 87	9/14 4/6	C1 C2	128 174	26 52	4/6 4/6	D1	354	78	3/5	-	-	-	-
3	Quartz Trachyte Porphyry	100	-	-	-	-	A1 A2	211 194	89 68	14/14 9/9	288 321	83 89	10/10 8/8	C1	091	25	4/4	-	-	-	-	-	-	-	-
4	Trachyte Breccia	29	-	-	-	-	A1	218	85	12/3	153	74	21/6	-	-	-	-	-	-	-	-	-	-	-	-
5	Hornblende Trachyte Porphyry	10	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	D1 D2	182 177	81 42	20/2 20/2	E1	083	88	20/2
6	Deadwood Sediments	11	FN	256	00	18/2	-	-	-	-	141	86	27/3	-	-	-	-	D1 D2	196 183	78 80	18/2 18/2	-	-	-	-

NOTES:

1. The main orientations of the discontinuity sets were determined from contoured lower hemisphere equal area projections of joints mapped in the structural areas.

2. "%/NO." refers to the percent concentration and corresponding number of joints in a one percent area of the lower hemisphere for the average or peak orientation of the population.

TABLE II

SUMMARY OF ROCK QUALITY DESIGNATION FROM DRILLHOLE LOGS

ROCK TYPE	LOCATION		RQD	
	DIAMOND DRILLHOLE	DEPTH INTERVAL (FT)	RANGE	AVERAGE
TRACHYTE PROPHYRY (OXIDE)	62	297 - 435	0 - 86	35.6
	63	125 - 144	———	33.0
TRACHYTE PORPHYRY (SULFIDE)	62	435 - 600	48 - 98	83.5
	63	543 - 600	46 - 78	61.7
	64	61.5 - 174	38 - 91	61.2
		259.0 - 289	80 - 100	91.3
QUARTZ TRACHYTE PROPHYRY (OXIDE)	62	9.0 - 294	0 - 86	41.8
TRACHYTE BRECCIA (SULFIDE)	63	144 - 179 460 - 543	27 - 81	62.0
PRECAMBRIAN SCHIST (SULFIDE)	63	190 - 370 373 - 460	7 - 94	46.9
	64	174 - 259	57 - 86	72.0
		289 - 455	33 - 100	74.3
		470 - 549 551 - 577	41 - 100	87.3

TABLE III

SUMMARY OF ESTIMATED UNCONFINED COMPRESSEIVE STRENGTHS[1]
BASED ON POINT LOAD INDEX TESTING

ROCK TYPE	LOCATION		NUMBER OF TESTS	ESTIMATED MEDIAN UNCONFINED COMPRESSIVE STRENGTH (psi)
	DIAMOND DRILLHOLE	DEPTH INTERVAL (FT)		
TRACHYTE PORPHYRY (OXIDE)	62	331.5 - 332.2	3	22,700 - 30,200
TRACHYTE PORPHYRY (SULFIDE)	62 64	495.0 - 495.7 513.0 - 513.6 535.5 - 536.0 267.8 - 268.4 285.3 - 285.9	13	15,300 - 20,400
QUARTZ TRACHYTE PORPHYRY (OXIDE)	62	73.0 - 73.6 91.5 - 92.0	7	12,100 - 16,100
QUARTZ TRACHYTE PORPHYRY (SULFIDE)	28	968 970 975 976 977	13	16,600 - 22,100
PRECAMBRIAN SCHIST	63	283.0 - 283.5 326.0 - 326.6 406.0 - 406.6	11 (Parallel to foliation) 10 (Perpendicular to foliation)	7,600 - 10,100 29,700 - 39,600

NOTE: 1. Unconfined compressive strengths are based on the median point load strength value for the number of tests performed. Individual test results are corrected for a standard diameter of 50mm and unconfined compressive strengths calculated in accordance with standard procedures established by the International Society for Rock Mechanics (i.e. based on the formula UCS = (18 to 24) Is_{50})

TABLE IV

SUMMARY OF WEDGE AND PLANE FAILURE KINEMATICS AND ASSOCIATED ALTERNATIVE SLOPE CONFIGURATIONS FOR SLOPES IN PRECAMBRIAN ROCKS

DESIGN SECTOR	SLOPE DIP DIRECTION (°)	PRINCIPAL KINEMATIC CONTROLS (PRINCIPAL FAILURE MODES CONSIDERED TO CONTROL BENCH STABILITY)				POSSIBLE SLOPE GEOMETRIES BASED ON KINEMATIC ANALYSIS OF WEDGE AND PLANE FAILURE[3, 4]												COMMENTS
		TYPE OF FAILURE	DISCONTINUITY SETS INVOLVED	INTENSITY/[1] IMPORTANCE	APPARENT PLUNGE[2] OR DIP OF FAILURE (°)	ANTICIPATED BREAKBACK AT[5] BENCH CREST FOR VARIOUS BENCH HEIGHTS, ASSUMING 90° DESIGN BENCH FACE ANGLE (ft)				TOTAL BERM WIDTHS FOR VARIOUS BENCH HEIGHTS (ft)				INTERRAMP SLOPE ANGLES FOR VARIOUS BENCH HEIGHTS (°)				
						40 ft	60 ft	80 ft	100 ft	40 ft	60 ft	80 ft	100 ft	40 ft	60 ft	80 ft	100 ft	
PC-000	000	Wedges	A/B	Weak/Moderate	70	7.3	10.9	14.6	18.2	32.3	38.9	46.6	53.2	51.1	57.0	59.8	62.0	
PC-030	030	Planes	A	Weak/Moderate	72	7.1	10.6	14.1	17.6	32.1	38.6	46.1	52.6	51.3	57.2	60.0	62.3	
PC-060	060	Planes	A	Weak/Moderate	79	7.1	10.6	14.1	17.6	32.1	38.6	46.1	52.6	51.3	57.2	60.0	62.3	
PC-090	090	Planes	A	Weak/Moderate	89	7.1	10.6	14.1	17.6	32.1	38.6	46.1	52.6	51.3	57.2	60.0	62.3	
PC-120	120	-	-	-	*	7.1	10.6	14.1	17.6	32.1	38.6	46.1	52.6	51.3	57.2	60.0	62.3	
PC-150	150	-	-	-	*	7.1	10.6	14.1	17.6	32.1	38.6	46.1	52.6	51.3	57.2	60.0	62.3	
PC-180	180	Planes	FN	Moderate	63	10.2	15.3	20.4	25.5	35.2	43.3	48.4	60.5	48.7	54.2	56.8	58.8	
PC-210	210	Planes	FN	Strong	44	40.0	60.0	80.0	100.0	-	-	-	-	44.0	44.0	44.0	44.0	It is assumed that all bench faces will break back to foliation. Thus, the optimum slope is one in which foliation is not undercut.
PC-240	240	Wedges	FN/B	Moderate	39	24.7	37.0	49.4	61.7	49.7	65.0	81.4	96.7	38.8	42.7	44.5	46.0	
PC-270	270	Wedges	FN/B	Moderate	42	22.2	33.3	44.4	55.5	47.2	61.3	76.4	90.5	40.3	44.4	46.3	47.9	
PC-300	300	Planes Wedges	B FN/B	Weak/Moderate Moderate	80 54	14.5	21.8	29.1	36.3	39.5	49.8	61.1	71.3	45.4	50.3	52.6	54.5	
PC-330	330	Planes Wedges	B A/B	Weak/Moderate Weak/Moderate	83 73	7.1	10.6	14.1	17.6	32.1	38.6	46.1	52.6	51.3	57.2	60.0	62.3	

NOTES:
1. Intensity/Importance refers to the relative degree of development of individual joint sets in a given failure mode and the estimated likelihood of excavated bench faces breaking back to or forming along this failure mode.
2. The dip or apparent plunge which is considered to control slope design is circled for each design sector. This angle has been used as the effective bench face angle and to calculate anticipated bench crest breakback in subsequent assessments of alternative possible slope geometries. "*" indicates no strong kinematic control is present.
3. See Fig. .. for definition of slope geometry parameters.
4. For 40, 60, 80 and 100 ft high benches, effective berm widths for access, rockfall catchment, etc. are assumed to be 25, 28, 32 and 35 ft, respectively.
5. Due to the generally discontinuous nature of Joint Sets A and B, anticipated bench breakback has been reduced by 50% in all design sectors except PC-210.

TABLE V

SUMMARY OF WEDGE AND PLANE FAILURE KINEMATICS AND

ASSOCIATED ALTERNATIVE SLOPE CONFIGURATIONS FOR SLOPES IN TRACHYTE PORPHYRY

DESIGN SECTOR	SLOPE DIP DIRECTION (°)	PRINCIPAL KINEMATIC CONTROLS (PRINCIPAL FAILURE MODES CONSIDERED TO CONTROL BENCH STABILITY)				POSSIBLE SLOPE GEOMETRIES BASED ON KINEMATIC ANALYSIS OF WEDGE AND PLANE FAILURE[3,4]: ANTICIPATED BREAKBACK AT BENCH CREST FOR VARIOUS BENCH HEIGHTS, ASSUMING 90° DESIGN BENCH FACE ANGLE (ft)				TOTAL BERM WIDTHS FOR VARIOUS BENCH HEIGHTS (ft)				INTERRAMP SLOPE ANGLES FOR VARIOUS BENCH HEIGHTS (°)				COMMENTS
		TYPE OF FAILURE	DISCONTINUITY SETS INVOLVED	INTENSITY/ IMPORTANCE[1]	APPARENT PLUNGE[2] OR DIP OF FAILURE (°)	40 ft	60 ft	80 ft	100 ft	40 ft	60 ft	80 ft	100 ft	40 ft	60 ft	80 ft	100 ft	
TP-000	000	Planes Wedges	D B/D A/B	Moderate Moderate Strong	77 76 78	10.0	15.0	20.0	25.0	35.0	43.0	52.0	60.0	48.8	54.4	57.0	59.0	
TP-030	030	Planes Wedges	D B/D	Moderate Moderate	81 71	13.8	20.7	27.5	34.4	38.8	48.7	59.5	69.4	45.9	50.9	53.4	55.2	
TP-060	060	Planes Wedges	A B/D	Moderate Moderate	83 70	14.6	21.8	29.1	36.4	39.6	49.8	61.1	71.4	45.3	50.3	52.6	54.5	Assumes moderately drained conditions.
TP-090	090	Planes Wedges	A A/C B/D	Strong Weak Moderate	86 63 75	10.7	16.1	21.4	26.8	35.7	44.1	53.4	61.8	48.3	53.7	56.3	58.3	Assumes moderately drained conditions.
TP-120	120	Wedges Wedges	A/C B/D	Weak Moderate	51 84	7.1	10.6	14.1	17.6	32.1	38.6	46.1	52.6	51.3	57.2	60.0	62.3	Assumes moderately drained conditions.
TP-150	150	Planes Wedges	C A/C	Weak/Moderate Weak	69 48	15.4	23.0	30.7	38.4	40.4	51.0	62.7	73.4	44.7	49.6	54.1	53.7	Assumes moderately drained conditions.
TP-180	180	Planes Planes	C A	Weak Strong	53 81	7.1	10.6	14.1	17.6	32.1	38.6	46.1	52.6	51.3	57.2	60.0	62.3	Assumes moderately drained conditions.
TP-210	210	Planes Wedges	C A/B	Weak Strong	69 74	11.5	17.2	22.9	28.6	36.5	45.2	54.9	63.6	47.6	53.0	55.5	57.5	Assumes moderately drained conditions.
TP-240	240	Wedges	A/B	Strong	71	13.8	20.7	27.5	34.4	38.8	48.7	59.5	69.4	45.9	50.9	53.4	55.2	Assumes moderately drained conditions.
TP-270	270	Planes Wedges	B A/B	Strong Strong	86 73	12.2	18.3	24.4	30.6	37.2	46.3	56.4	65.6	47.1	52.3	54.8	56.7	Assumes moderately drained conditions.
TP-300	300	Planes Wedges	B A/B	Strong Strong	81 79	7.8	11.7	15.6	19.4	32.8	39.7	47.6	54.4	50.6	56.5	59.2	61.5	Assumes moderately drained conditions.
TP-330	330	Planes Wedges	D A/D A/B	Moderate Moderate Strong	86 77 78	9.2	13.9	18.5	23.1	34.2	41.9	50.5	58.1	49.5	55.1	57.7	59.8	Assumes moderately drained conditions.

NOTES:
1. Intensity/Importance refers to the relative degree of development of individual joint sets in a given failure mode and the estimated likelihood of excavated bench faces breaking back to or forming along this failure mode.
2. The dip or apparent plunge which is considered to control slope design is circled for each design sector. This angle has been used as the effective bench face angle and to calculate anticipated bench crest breakback in subsequent assessments of alternative possible slope geometries. "*" indicates no strong kinematic control is present.
3. See Fig. .. for definition of slope geometry parameters.
4. For 40, 60, 80 and 100 ft high benches, effective berm widths for access, rockfall catchment, etc. are assumed to be 25, 28, 32 and 35 ft, respectively.

TABLE V

TABLE VI

SUMMARY OF WEDGE AND PLANE FAILURE KINEMATICS AND ASSOCIATED ALTERNATIVE SLOPE CONFIGURATIONS FOR SLOPES IN QUARTZ TRACHYTE PORPHYRY

		PRINCIPAL KINEMATIC CONTROLS (PRINCIPAL FAILURE MODES CONSIDERED TO CONTROL BENCH STABILITY)				POSSIBLE SLOPE GEOMETRIES BASED ON KINEMATIC ANALYSIS OF WEDGE AND PLANE FAILURE[3, 4]												
						ANTICIPATED BREAKBACK AT BENCH CREST FOR VARIOUS BENCH HEIGHTS, ASSUMING 90° DESIGN BENCH FACE ANGLE (ft)				TOTAL BERM WIDTHS FOR VARIOUS BENCH HEIGHTS (ft)				INTERRAMP SLOPE ANGLES FOR VARIOUS BENCH HEIGHTS (°)				
DESIGN SECTOR	SLOPE DIP DIRECTION (°)	TYPE OF FAILURE	DISCONTINUITY SETS INVOLVED	INTENSITY/ IMPORTANCE[1]	APPARENT PLUNGE[2] OR DIP OF FAILURE (°)	40 ft	60 ft	80 ft	100 ft	40 ft	60 ft	80 ft	100 ft	40 ft	60 ft	80 ft	100 ft	COMMENTS
QTP-000	000	Wedges	A/B	Strong	87	7.1	10.6	14.1	17.6	32.1	38.6	46.1	52.6	51.3	57.2	60.0	62.3	
QTP-030	030	Planes	C	Weak	*	7.1	10.6	14.1	17.6	32.1	38.6	46.1	52.6	51.3	57.2	60.0	62.3	Assumes moderately drained conditions.
QTP-060	060	Planes	C	Weak	*	7.1	10.6	14.1	17.6	32.1	38.6	46.1	52.6	51.3	57.2	60.0	62.3	Assumes moderately drained conditions.
QTP-090	090	Planes	C	Weak	*	7.1	10.6	14.1	17.6	32.1	38.6	46.1	52.6	51.3	57.2	60.0	62.3	Assumes moderately drained conditions.
QTP-120	120	Planes Wedges	C A/C	Weak Weak	*	7.1	10.6	14.1	17.6	32.1	38.6	46.1	52.6	51.3	57.2	60.0	62.3	Assumes moderately drained conditions.
QTP-150	150	Wedges	A/C	Weak	*	7.1	10.6	14.1	17.6	32.1	38.6	46.1	52.6	51.3	57.2	60.0	62.3	Assumes moderately drained conditions.
QTP-180	180	Wedges	A/C B/B	Weak Moderate	83	7.1	10.6	14.1	17.6	32.1	38.6	46.1	52.6	51.3	57.2	60.0	62.3	Assumes moderately drained conditions.
QTP-210	210	Wedges	B/B A/B	Moderate Strong	79 86	7.8	11.7	15.6	19.4	32.8	39.7	47.6	54.4	50.6	56.5	59.2	61.5	
QTP-240	240	Wedges	B/B A/B	Moderate Strong	78 83	8.5	12.8	17.0	21.3	33.5	40.8	49.0	56.3	50.1	55.8	58.5	60.6	
QTP-270	270	Wedges	B/B A/B	Moderate Strong	80 82	7.1	10.6	14.1	17.6	32.1	38.6	46.1	52.6	51.3	57.2	60.0	62.3	
QTP-300	300	Wedges	A/B	Strong	82	7.1	10.6	14.1	17.6	32.1	38.6	46.1	52.6	51.3	57.2	60.0	62.3	
QTP-330	330	Wedges	A/B	Strong	84	7.1	10.6	14.1	17.6	32.1	38.6	46.1	52.6	51.3	57.2	60.0	62.3	

NOTES:
1. Intensity/importance refers to the relative degree of development of individual joint sets in a given failure mode and the estimated likelihood of excavated bench faces breaking back to or forming along this failure mode.
2. The dip or apparent plunge which is considered to control slope design is circled for each design sector. This angle has been used as the effective bench face angle and to calculate anticipated bench crest breakback in subsequent assessments of alternative possible slope geometries. "*" indicates no strong kinematic control is present.
3. See Fig. .. for definition of slope geometry parameters.
4. For 40, 60, 80 and 100 ft high benches, effective berm widths for access, rockfall catchment, etc. are assumed to be 25, 28, 32 and 35 ft, respectively.

TABLE VII(a)

SUMMARY OF THE EFFECT ON INTERRAMP SLOPE ANGLES OF FLATTENTING THE EFFECTIVE BENCH FACE ANGLE[1]

ACTUAL BENCH FACE ANGLE (°)	INTERRAMP SLOPE ANGLE FOR VARIOUS BENCH HEIGHTS (°)			
	40 ft	60 ft	80 ft	100 ft
80	51.3	57.3	60.0	62.2
75	48.2	53.7	56.3	58.3
70	45.3	50.3	52.6	54.5
65	42.5	47.0	49.1	50.8

NOTE: 1. Effective berm widths of 25, 28, 32 and 35 feet are assumed to be maintained for 40, 60, 80 and 100 foot high benches.

TABLE VII(b)

SUMMARY OF THE EFFECT ON BERM WIDTH OF FLATTENING AN INITIAL 80° BENCH FACE ANGLE[1]

ACTUAL BENCH FACE ANGLE (°)	REMAINING EFFECTIVE BERM WIDTH FOR VARIOUS BENCH HEIGHTS (FT)			
	40 ft	60 ft	80 ft	100 ft
80	25	28	32	35
75	21.3	22.5	24.7	25.8
70	17.5	16.7	17.0	16.2
65	12.0	10.6	8.8	6.0

NOTE: 1. Initial effective berm widths of 25, 28, 32 and 35 feet are assumed for 40, 60, 80 and 100 feet high benches, respectively.